DETAILED PROJECT REPORT
WITH ENVIRONMENTAL ASSESSMENT

SECTION 205 FLOOD DAMAGE REDUCTION STUDY

MAD CREEK
MUSCATINE, MUSCATINE COUNTY, IOWA

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HYDROLOGY AND HYDRAULICS

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1. PURPOSE AND SITE DESCRIPTION

This appendix documents efforts to reduce flood damage in the City of Muscatine from flooding by the Mississippi River and by Mad Creek. The City of Muscatine is on the Mississippi River approximately 1 mile downstream from Lock and Dam 16. A location map of the study area appears on Figure 1 of the Detailed Project Report showing retention (basin) locations. A vicinity and location map along with the retention locations can be found on the plates, Appendix L. Mad Creek (drainage area about 17 square miles) starts in the hills above Muscatine, flows through the city, and enters the Mississippi River at river mile 546. Geneva Creek, a tributary to Mad Creek, has a drainage area of 3 square miles. Geneva Creek flows southwesterly into the city and enters Mad Creek; this confluence is in a commercial area of the city. Mad Creek continues downstream about 6,000 feet to its mouth. Storm water from a portion of the city, including residential and downtown areas, does not contribute flow to Mad Creek due to an existing levee system.

2. CLIMATE, FLOODS OF RECORD, AND DATUM

The temperature summary at Muscatine (National Weather Service station 5837) is based on the period from 1948 to 1999. The average monthly temperature ranges from 22 degrees Fahrenheit (January) to 76 degrees Fahrenheit (July). The mean daily temperature is 51 degrees Fahrenheit. Extremes include two days between 105 to 114 degrees Fahrenheit and one day between -40 to -31 degrees below zero.

Peak discharges are not recorded on Mad Creek. Large floods occurred on June 30, 1961, and on June 16, 1990. The gage at Muscatine on the Mississippi River is at mile 453. The 10 highest recorded stages at this gage since 1878 appear in Table A-1. The gage zero is elevation 530.74 feet MSL.

Table A-1. Peak Stages on the Mississippi River at Muscatine, Iowa

Rank	Date	Peak Stage
1	07/09/1993	25.61
2	04/29/1965	24.81
3	04/251973	21.63
4	04/26/1969	21.20
5	04/21/1997	21.09
6	04/28/1952	21.05
7	04/26/1951	21.00
8	05/09/1975	20.96
9	10/07/1986	20.59
10	04/16/1967	19.40

More that one datum exists at Muscatine. Measurements of stage on the Mississippi River are in MSL (1912 datum). This datum was used for earlier reports and drawing of levees along the Mississippi River. Elevations on USGS quad sheets are in NGVD (1929 datum). At Muscatine, subtracting 0.49 foot from an elevation in 1912 datum converts it to an elevation in 1929 datum. The City of Muscatine uses a city datum. Adding 249.1 feet to an elevation in city datum converts it to an elevation in 1929 datum. Elevations in this appendix are in or have been converted to 1929 datum.

3. EXISTING FEDERAL PROJECTS

Two Federal projects are located within the City of Muscatine. The design stage appears to have been 3 feet higher than the 1% probability event at the time of design for both projects. (Refer to plates X101 thru X105 in Appendix L for project maps.) The oldest system was finished in 1961 and improved in 1983; it protects downtown Muscatine from flooding by the Mississippi River and by Mad Creek. The protected area is triangular-shaped. One levee is along the Mississippi River (460 feet); the other is along the right bank of Mad Creek (3,000 feet). A second levee system was finished in 1983 and mainly protects the Heinz plant where Geneva Creek enters Mad Creek. Geneva Creek enters Mad Creek about 6,000 feet upstream from the mouth of Mad Creek. Stages for the Mississippi River have increased since the 1961 report.

4. PROPOSED FEDERAL ALTERNATIVES

Four alternatives to reduce flood damage are examined in this report. Alternative A proposes raising the existing levees along Mad Creek, improving the closure structures, and improving the flood warning system. Alternative B proposes building one storm water detention reservoir on Geneva Creek and one storm water detention reservoir on Mad Creek. The reservoirs will only store water during a flood event. Alternative C examines a combination of reservoirs and levee raise. Alternative D proposes raising levees on both Mad Creek and the Mississippi River, plus improving the channel, the closure structures, and the flood warning system.

The Corps of Engineers also plans to model several alternatives for the City of Muscatine to evaluate raising the 5th Street Bridge and the roadway within the floodway of Mad Creek.

5. PREVIOUS HYDROLOGIC STUDIES OF MAD CREEK BY THE CORPS OF ENGINEERS

Mad Creek has no recording stream gage. The Rock Island District has developed discharge-frequency relationships for Mad Creek on several occasions. The first hydrologic study by the District was made in the 1950's (reference 1). The peak discharge for the 1% chance event was 4,900 cfs. This discharge was increased during the 1960's for the existing Federal projects. The discharge-frequency relationship was based on a flood flow frequency analysis of Mill Creek in Milan, Illinois (drainage area 62.5 sq. mi.). A representative standard deviation (S) of 0.342 and the log of the mean annual flood (M) 3.142 were used to compute the discharge-frequency curve. The log of the mean annual flood for Mad Creek was based on a ratio of the drainage areas (references 2 and 3). This methodology is no longer the preferred method of the Corps of Engineers.

Discharges used in a 1977 flood insurance study (reference 4) were based on a Corps of Engineers regression analysis of bluff streams along the Mississippi River. The discharges were from "expected probability discharges" equations, which give larger values than "computed probability discharges."

In 1996, the NRCS (Natural Resources Conservation Service) made a basin model for the City of Muscatine, which evaluated many different dam sites (reference 5). However, the loss rates and unit-hydrograph parameters in this model were not verified on a similar, gaged basin. In 2000, the Corps of Engineers developed an HEC-HMS (reference 6) model of Mad Creek. This model used the two dams from the NRCS model that would be most likely to reduce flood damage. The sites were coordinated with the local sponsor. The loss rates in the HEC-HMS model were verified on a gaged basin; however, insufficient rainfall data prevented calibrating any unit hydrograph parameters. The engineers at the Corps' Hydrologic Engineering Center at Davis, California, prefer the regression analysis for computing the absolute value of discharges. HEC-HMS is good for evaluating variations in discharge, but is not recommended for determining the absolute value of discharge. The without-project discharges from the future condition (year 2020) computed by the HEC-HMS model appear in Tables A-2 and A-3. Discharge-frequency data that were available for the other methods appear in the same tables. "RI" stands for recurrence interval in years.

Table A-2. Comparison of discharges on Mad Creek

Discharge Probability %	RI Yr.	Mouth of Mad 1960's cfs	Mouth of Mad FIS cfs	Mouth of Mad HMS cfs	U/S Conf. Geneva 1960's cfs	U/S Conf. Geneva FIS cfs	U/S Conf. Geneva HMS cfs
50	2	1,400		1,581			1,125
20	5	2,700		3,135			2,296
10	10	4,000	3,140	4,252		2,780	3,135
4	25	6,100		5,638			
2	50	7,900	6,100	7,613		5,370	5,480
1	100	10,200	7,700	8,733	6,600	6,820	6,525
0.4	250	14,500		12,093			
0.2	500	17,000	12,100	18,327		10,800	13,610
Mi^2		17.3	17.50	16.93	13.8	14.00	13.33

Table A-3. Comparison of discharges, Mouth of Geneva Creek

Discharge Probability %	Geneva 1960's cfs	Geneva FIS cfs	Geneva HMS cfs
50			468
20			906
10		1,140	1,222
4			1,615
2		2,330	2,110
1	3,600	3,020	2,487
0.4			3,426
0.2		4,960	5,090
Mi^2	2.9	2.90	3.05

6. MISSISSIPPI RIVER HYDROLOGY AND HYDRAULICS

Discharge data for the Mississippi River at Muscatine and river stages at the mouth of Mad Creek appear in Table A-4. These data are from a report prepared by the Rock Island District in 1979 for the Technical Flood Plain Management Task Force of the Upper Mississippi River Basin Commission and used for studies on the Mississippi River (reference 7). Data in the original report were converted to 1929 datum for this table. Some points were interpolated.

Table A-4. Discharge and stage, Mississippi River at Muscatine, Iowa (NGVD 1929)

Probability %	Discharge cfs	Stage ft NGVD
50	153,000	548.10
20	203,000	550.46
10	235,000	552.21
4	273,000	554.65
2	308,000	556.26
1	335,000	557.50
0.4	370,000	559.20
0.2	400,000	560.36

7. HYDROLOGIC MODELING

Daily and Associates of Peoria, Illinois, prepared the HEC-HMS modeling and written summary under contract to the Corps of Engineers. The original report (reference 8) is on file at the Rock Island District office.

a. <u>HEC-HMS Without-Project and With-Project Modeling</u>. Two HEC-HMS (reference 6) models were prepared for this study. The first modeled the existing basin. The "without-project model" was used to evaluate the existing levees. Moreover, since raising levees does not alter the discharge-frequency relationship, it was also used to evaluate Alternatives A and D. The "with-project model" was used to evaluate Alternatives B and C (alternatives with water detention reservoirs).

All HEC-HMS models used the SCS Type II rainfall distribution with a 24-hour duration. This distribution produces the highest peak flows. The synthetic rainfall amounts came from Bulletin 71 (reference 9). Discharges were computed at the mouth of Mad Creek, the mouth of Geneva Creek, and on Mad Creek upstream of the confluence of Geneva Creek.

(1) <u>Gimlet Creek at Sparland Used to Check Loss Rates</u>. Since Mad Creek is ungaged, loss rates were verified using an HEC-HMS model of Gimlet Creek at Sparland, Illinois. This basin with similar soils and terrain is located approximately 41 degrees latitude along the west bank of the Illinois River. Gimlet Creek, USGS 05559000, has a drainage area of 5.66 square miles, a slope of 53.86 feet per mile, and a channel length of 4.81 miles. The gage recorded 36 annual peak discharges in years 1924, 1946, 1947, and 1950-1982. Discharges were analyzed using a flood flow frequency analysis and are summarized in Table A-5. The 24-hour rainfall for Gimlet Creek came from Bulletin 71 for the central region of Illinois (see Table A-5).

Table A-5. Gimlet Creek flood flow frequency results and rainfall

Expected Probability (%)	Peak Discharge cfs	Storm Probability	Rainfall in Inches
50	809	0.500	3.02
20	1,280	0.040	5.32
10	1,580	0.010	6.92
5	1,870		
2	2,230		
1	2,500		
0.2	3,100		

The Clark unit hydrograph transformed rainfall excess into runoff. The time of concentration was calculated for each subbasin by breaking the total channel length into parts and estimating velocity and travel time for each segment. The storage coefficient was estimated using reference 10. For the region containing Gimlet Creek, the storage coefficient equals the time of concentration.

The Muskingum method was used for channel routing. The routing has two parameters: (1) travel time (K) through the routing reach, and (2) a dimensionless constant (X). If X is 0.0, the maximum attenuation occurs; if X is 0.5, no attenuation occurs. The X value was based on experience. X approaches 0.0 if the channel has mild slopes with flows out of banks and approaches 0.5 for well-defined channels where the discharge stays within banks. The travel time was estimated with Manning's equation and typical cross-sectional geometry.

The Green and Ampt parameters used to determined rainfall losses appear in Table A-6. The moisture deficit is the antecedent moisture condition, the wetting front suction measures the ability of soil to draw water into the ground before saturation, and hydraulic conductivity measures the rate water passes through soil. Values were not from physical tests, but estimated from soil type (silt loams to silty clay loams). The computed peak at Gimlet Creek for the 100-year, 24-hour storm was 2,510 cfs. It was obtained by varying parameters within acceptable ranges so the calculated peak was nearly equal to the flood flow frequency peak. The parameters were then used for all frequency storms. The impervious area was estimated from Quad maps.

Table A-6. Loss rates derived from Gimlet Creek for HEC-HMS models

Initial Loss (Inches)	Volumetric Moisture Deficit	Wetting Front Suction (Inches)	Hydraulic Conductivity (In/Hr)
.05-.1	.22	8	.5

(2) <u>Mad Creek Without-Project Model</u>. The regional charts in Bulletin 71 show that Muscatine receives greater rainfall than the rest of its region. Regional maps, not regional tables, were used for synthetic rainfall (Table A-7). The partial-series rainfall amounts for the 50% through 10% probability events were converted to annual series for this study. Since synthetic rainfall for 99%, 0.4%, and 0.5% probability events is not published, values were extrapolated. The extrapolated data were used for informational purposes only.

Table A-7. Rainfall used at Muscatine for 24-hour storms

Probability %	Partial Series Rainfall Inches	Annual Series (Adjusted) Inches
99.9	2.9	
50	3.2	2.83
20	4.0	3.80
10	4.6	4.55
4	5.5	
2	6.8	
1	7.5	
0.4	9.6	
0.5	13.1	

(3) <u>Subbasin Parameters</u>. The basin map of Mad Creek appears on plate A-1, while the schematic of the HEC-HMS With-Project model appears in Figure A-1. The Clark time of concentration (Tc) and the Clark R-value calculated for each subbasin appear in Table A-8. The computation method was described in the paragraph on Gimlet Creek. All models used the same Green and Ampt loss parameters calibrated from Gimlet Creek (see Table A-6).

Table A-8. Mad Creek HEC-HMS subbasin parameters

Subbasin ID	Drainage Area Sq. Mi.	Clark Tc Hours	Clark R Hours	Impervious Percent Yr. 2000	Impervious Percent Yr. 2020
1	7.06	1.93	1.93	2	5
2	4.58	1.08	1.08	2	5
3	1.69	1.13	1.13	10	20
4	2.09	1.07	1.07	2	6
5	0.96	0.87	0.87	18	24
6	0.55	0.92	0.92	35	35

Initially, discharge-frequency relationships were computed for the year 2000 and for the year 2020. The city engineer predicted development adjacent to Highway 38 and the Highway 61 Bypass. The impervious percent was increased (see Table A-8) to reflect future development. The contractor did not believe Tc and R would change as a result of development. The difference in discharges was so slight that only year 2020 discharges were used in this study.

The coefficients used for Muskingum routing through sub-basins appear in Table A-9. The number of sub-reaches depends upon the computation interval, which was 15 minutes.

Table A-9. Muskingum routing values used in HEC-HMS

Reach ID	Muskingum K Hrs.	Muskingum X	Number of Sub-Reaches
Reach 1	.50	.15	1
Reach 2	.40	.15	1
Geneva 3	.45	.20	1
Mad 4	.35	.15	1

(4) <u>Mad Creek With-Project Model</u>. Two Modified Puls routings added to the without-project model simulated the storm water detention reservoirs in the with-project model (see Figure A-1). Dam 2 on Mad Creek is just outside the current city limit. Existing sewage disposal lagoons on one of the tributaries determined the guide for the limiting pond elevation. Since the crest of the lagoon is elevation 636.4 feet NGVD, operation would be restricted at a lower elevation. Dam 1 is on Geneva Creek (just outside of the current city limit). The elevation of Highway 61 Bypass served as the guide for limiting pond elevation.

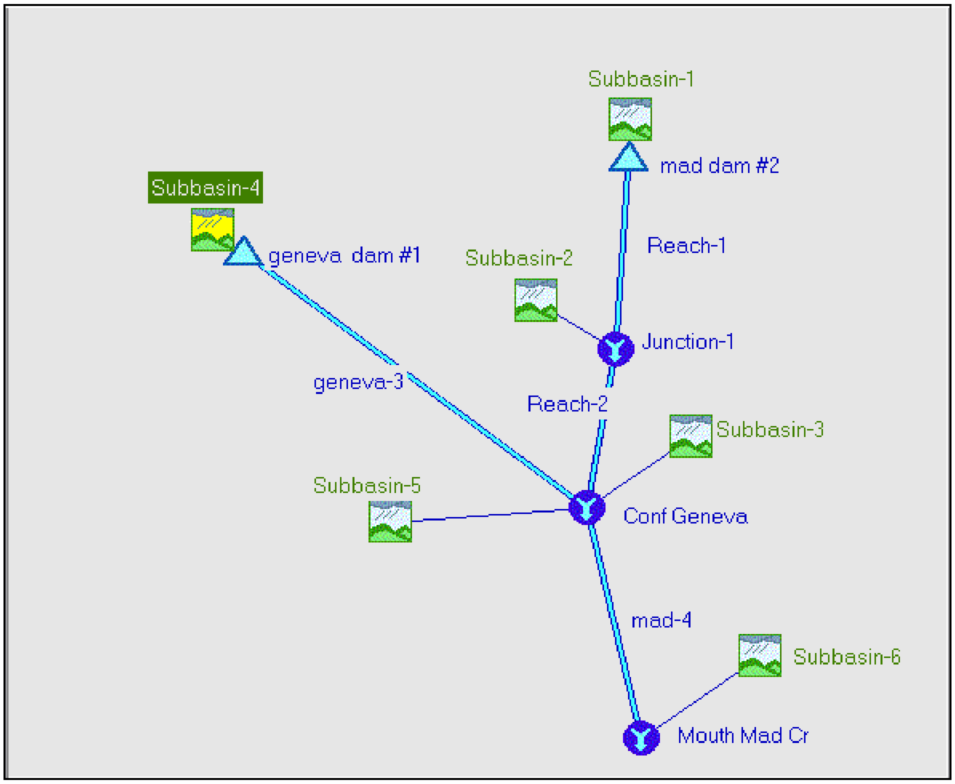

Figure A-1. Schematic diagram of with-project model

Only about half of the total area of Mad Creek is upstream of the proposed storm water detention reservoirs. To influence the discharges within the city, the peak flows at the reservoirs must decrease significantly. The selected outlet consisted of a circular culvert and an emergency spillway. The weir coefficient for the broad-crested emergency spillway was about 2.65.

The primary outlet of the Mad Creek Dam is 48 inches in diameter at the existing flowline of the creek. The crest of the emergency spillway is at elevation 636.5 feet; the weir length is 250 feet. The primary outlet of the Geneva Creek Dam is 36 inches in diameter. It is set in the flowline of the existing creek. The crest of the emergency spillway is elevation 629 feet; the weir length is 125 feet. Data (elevation, area, and outflow) used to model the reservoirs appear in Table A-10.

Table A-10. Elevation-area-outflow data for the proposed reservoirs

Elevation Ft NGVD	Mad Area Ac	Mad Outflow cfs	Elevation Ft NGVD	Geneva Area Ac	Geneva Outflow cfs
602	0	0	600	0	0
610	6.49	164.82	610	5.30	115.61
620	35.67	285.58	620	18.21	174.57
630	73.92	368.29	629	35.83	213.78
636.5	109.99	412.02	630	37.79	594.14
637	112.76	649.38	631	40.76	1,158.76
638	118.30	1,639.11	631.5	42.24	1,532.57
639	123.85	3,258.00	635	51.00	5,104.00
640	129.40	4,773.56	640	67.44	12,339.36
641	134.95	6,766.29	650	124.97	32,163.56

b. <u>HEC-HMS Results from Synthetic Storms and Recommended Discharges</u>. Table A-11 summarizes the influence of the proposed reservoirs. The table lists the computed peak inflow and outflow (year 2020). The table also lists the maximum impoundment elevation calculated for each probability event. Most elevations were rounded to the nearest foot.

Table A-11. Computed inflow, outflow, and peak water surface elevation at proposed reservoirs

Storm Probability %	Mad Creek Reservoir			Geneva Creek Reservoir		
	Inflow cfs	Outflow cfs	Stage Ft NGVD	Inflow cfs	Outflow cfs	Stage Ft NGVD
50	625	204	613	320	119	611
20	1,323	286	620	674	140	614
10	1,823	302	622	928	157	617
4	2,443	324	625	1,242	176	620
2	3,331	356	629	1,682	186	622
1	3,839	371	630	1,930	192	623
0.4	5,352	398	635	2,665	209	628
0.2	8,116	2,212	638.35	3,957	1,192	631.04

The peak discharges computed at the mouth of Mad Creek and the mouth of Geneva Creek appear in Table A-12. These data produced an erratic line when plotted on discharge-frequency paper. To eliminate problems the scatter could cause when used in the HEC-FDA model, the data were fitted to a curved line. The adjusted data used in this study appear in Table A-13. A plot of discharge-frequency from Tables A-12 and A-13 for the mouth of Mad Creek appears on plate A-2. A similar plot for Geneva Creek is on plate A-3.

The difference between year 2000 and 2020 peak discharges was in the range of 1% to 5%. The larger the peak discharge, the less the percent difference. The influence of future development was so small that the future discharges were used for both present and future conditions to evaluate alternatives.

Table A-12. Preliminary HEC-HMS discharges, mouth of Mad Creek and mouth of Geneva Creek

Probability %		Mad Without Reservoirs Yr. 2020 cfs	Mad With Reservoirs Yr. 2020 cfs		Geneva Without Reservoirs Yr. 2020 cfs	Geneva With Reservoirs Yr. 2020 cfs
50		1,580	1,240		468	286
20		3,130	2,335		910	501
10		4,250	3,100		1,220	640
4		5,638	4,053		1,615	825
2		7,613	5,388		2,173	1,069
1		8,733	6,137		2,487	1,201
0.4		12,093	8,371		3,426	1,597
0.2		18,327	12,371		5,095	2,285

Table A-13. Adopted discharges for mouth of Mad Creek and mouth of Geneva Creek

Probability %		Mad Without Reservoirs Yr. 2020 cfs	Mad With Reservoirs Yr. 2020 cfs		Geneva Without Reservoirs Yr. 2020 cfs	Geneva With Reservoirs Yr. 2020 cfs
50		1,580	1,240		393	286
20		2,880	2,200		797	501
10		3,974	3,000		1,188	640
4		5,636	4,300		1,798	840
2		7,089	5,400		2,356	1,069
1		8,733	6,600		3,010	1,201
0.4		11,240	8,400		4,043	1,700
0.2		13,411	10,000		4,968	2,285

c. <u>HEC-HMS Results from Probable Maximum Storm</u>. Both proposed storm water detention reservoirs are upstream of the City of Muscatine. Since the failure of either dam could result in loss of life and property, the dams would be classed as high hazard. The State of Iowa requires such reservoirs to be evaluated using the probable maximum storm. This storm is the most extreme rainfall possible at the site. A probable maximum storm was routed through the with-project HEC-HMS model to evaluate the performance of the reservoirs under this extreme event. A point was added to the routing of Mad Creek reservoir for elevation 645 feet (200 acres and 16,888 cfs outflow). At the Geneva Creek reservoir, computations showed: inflow 7,360 cfs, outflow 6,340 cfs, and pond elevation 635.8 feet NGVD. At Mad Creek, computations showed: peak inflow 18,600 cfs, outflow 15,750 cfs, and pond elevation of 644.6 feet NGVD. For purposes of comparison to synthetic events, the peak discharge at the mouth of Mad Creek (with-project) was 32,700 cfs.

8. HYDRAULIC MODELING

Water surface profiles were computed on HEC-RAS (river analysis system, V 2.2, reference 11). The lower channel has not changed since the construction of previous projects by the Corps of Engineers. Cross sections in these areas were taken from Corps of Engineers drawings and city topographic maps. However, upper portions have re-aligned and the floodway has filled. Sixteen cross sections were surveyed between the Route 61 Bypass and Washington Street.

Manning's N-values was based upon judgement. The N-value for the main channel ranged from .035 to .036 while the overbank N-values ranged from .04 to .07 for overbanks covered with grass to brush.

The lower Mad Creek model ran from its mouth upstream to the junction of Geneva Creek, about 6,000 feet. The model had 5 bridges, 42 cross sections, and started at normal depth. Discharges from Table A-13 were used in the HEC-RAS model.

The Geneva Creek model started at its mouth and ran upstream 3,000 feet. The model had two bridges, six cross sections, and started with known water-surface levels. The starting elevations were at the junction of Geneva Creek from the lower Mad Creek model for the equivalent frequency and alternative (see Table A-14).

Table A-14. Starting water surface elevations at the mouth of Geneva Creek

Probability %	Without Project WSEL* Feet	Without Project Discharge cfs		With Project WSEL Feet	With Project Discharge cfs
50	555.23	393		554.50	286
20	557.47	797		556.37	501
10	559.00	1,188		557.65	640
4	560.99	1,798		559.41	840
2	562.47	2,356		560.74	1,069
1	563.93	3,010		561.96	1,201
0.4	565.75	4,043		563.65	1,700
0.2	567.16	4,968		564.91	2,285
* water surface elevation					

In preparing the without-project model, engineers noticed a sandbar blocking half of the 2nd Street Bridge. At the upstream face of the bridge, the sand extended from the center pier to the north bridge abutment. Since it was not certain the sand would wash away, a third model was prepared (without-project improved channel). This model was used to evaluate Alternative D. The sand was removed from 2nd Street, and the channel upstream and downstream of the bridge was widened (bottom width of 45 feet with 1 on 3 side slopes).

HEC-RAS Model Results. The without-project profiles modeled lower Mad Creek using future condition discharges and the present channel. These profiles were used to evaluate the existing levee system and Alternative A (levee raise) (see plate A-4). The with-project profiles modeled the proposed reservoirs and the present channel. These profiles were used to evaluate Alternative B and Alternative C on Mad Creek and appear on plate A-5. Profiles for Alternative D used future condition discharges and an improved channel; profiles appear on plate A-6. Figure A-2 superimposes the 10% exceedance event for Alternatives A (no reservoirs), B (with reservoirs), and D (no reservoirs but improved channel). Alternative B eliminates the constriction at 2nd Street by lowering the discharge; Alternative D eliminates the constriction by increasing the bridge opening.

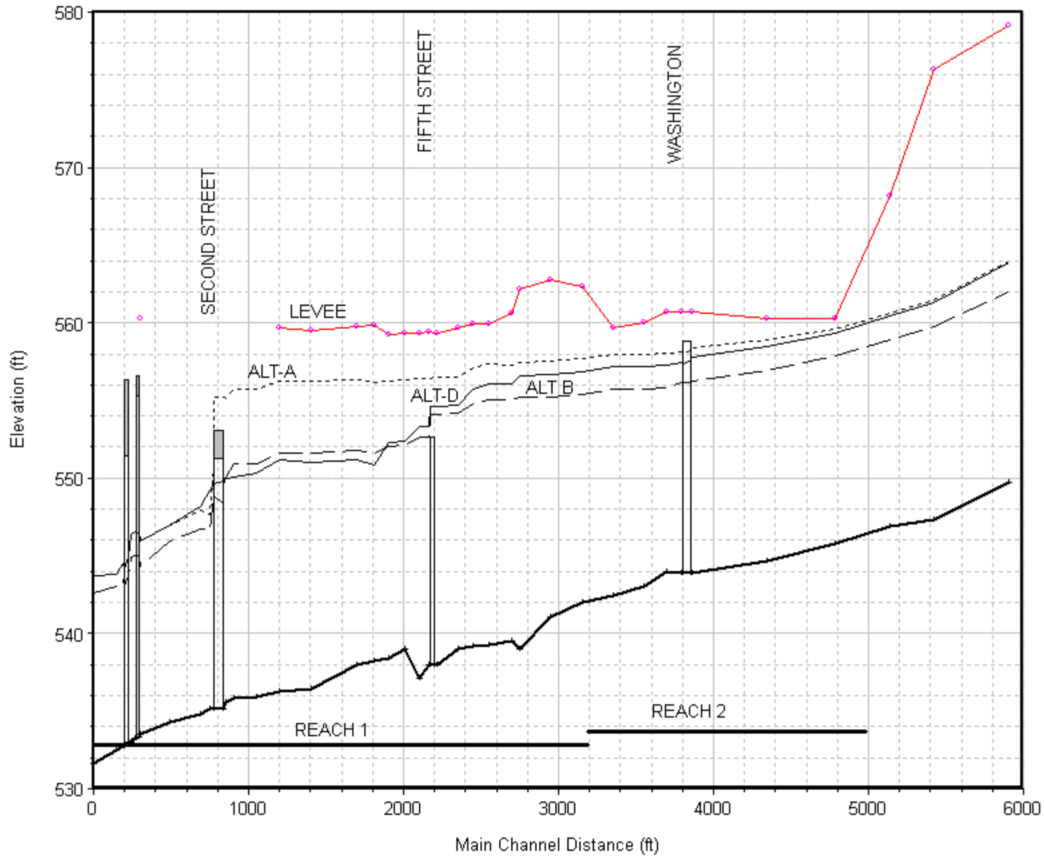

Figure A-2. Profiles of alternatives on Mad Creek for .01 probability event

The improved channel lowered the water surface profiles from 2nd Street to above Washington. While the improvement lowers the water level, it will not eliminate the sediment problem at the bridge. Maintenance cleaning will be required periodically. For purposes of estimating maintenance costs, cleaning will be required every 2 to 5 years.

The without-project profiles for Geneva Creek appear on plate A-7, and the with-project profiles appear on plate A-8. The improved channel at 2nd Street does not lower the starting water level on Geneva Creek significantly. The decrease in water levels for various events ranged from 0.00 to 0.06 foot. For a map showing the locations of HEC-RAS cross sections, refer to plate A-9.

9. RISK BASED ANALYSIS

This section discusses the hydrologic and hydraulic input used for HEC-FDA (flood damage reduction analysis program, reference 12). After a general discussion on input, the performance statistics for the alternatives are presented by economic reach.

a. <u>Description of HEC-FDA Input</u>.

(1) <u>Discharge Frequency</u>. The HEC-FDA computer program (Version 1.2, March 2000) was used to evaluate alternatives. The Rock Island District's Economic and Social Analysis Section identified four reaches at Muscatine. Reach 1 examined damages in downtown

Muscatine from flooding by Mad Creek. The reach started at the mouth and ended upstream of 5th Street. Reach 2, also on Mad Creek, started upstream of 5th Street and ended just upstream of Washington Street. Reach 3 evaluated damages behind the levees near the confluence of Geneva Creek and Mad Creek. Reach 4 also examined damages in downtown Muscatine, but this time from flooding by the Mississippi River. For a discussion of the reaches and a map, refer to Appendix B - Economic Analysis.

Hydrologic input included discharge-frequency relationships and variation in discharge for each reach. Hydraulic input consisted of a stage-discharge relationship and an estimate of the standard error of stage in feet. Levee input included the crest elevation of existing or proposed levees. Flooding on the Mississippi River and Mad Creek were analyzed as independent events. Flooding on Geneva Creek and Mad Creek were assumed to be concurrent.

The analytical option in HEC-FDA created the without-project discharge-frequency curves. Discharges for the 50%, 10% and 1% exceedance-probability events were entered to generate discharge-frequency curves. For Mad Creek and Geneva Creek, these discharges appear in Table A-12. A period of record of 20 years was used on both creeks to generate confidence limits. The equivalent record length for HEC-HMS models using regional model parameters is 10 to 30 years, so the midpoint was used (reference 13). The without-project discharge-frequency data were used for Alternatives A and D. For the Mississippi River, the appropriate discharges from Table A-4 were used with a 70-year period of record. Seventy years is the average record period of the gages used to determine the discharges at Muscatine appearing in Table A-4 from reference 7. These discharge-frequency data were used for Alternative D.

Since the with-project discharges are regulated, the graphical instead of the analytical option was required for Mad Creek. To create the with-project discharge-frequency relationship, the discharges from the without-project HEC-FDA curve (the 99.9%, 50%, 10%, 1%, 0.4% and 0.2% exceedance probability events) were entered into the graphical option. However, a period of record of 10 years instead of 20 was used to generate confidence limits. The shorter period was used so that the confidence limits from both the analytical and graphical methods produced similar economic results for the without-project alternative.

Once this was established, a transform feature in HEC-FDA converted the unregulated discharges to regulated discharges. For Mad Creek, the transform correlated each without-project discharge to a with-project discharge. The variation in with-project discharges was described with a triangular distribution that listed minimum and maximum values for each entry in the transform table. This variation quantified the error in discharge at the mouth of Mad Creek attributable to the two upstream reservoirs. Variations were determined by estimating the variation in culvert discharge and, if applicable, the variation in spillway discharge. The discharge from the culvert was varied by plus or minus 20%. The variation in spillway discharge was calculated by varying the spillway weir coefficient from 2.63 to 3.087. The without-project discharge-frequency relationship evaluated Alternatives B and C in Reaches 1 and 2.

(2) <u>Stage Discharge</u>. HEC-RAS profiles provided the stage-discharge data for Mad Creek and Geneva Creek. A standard error in stage of 1 foot was assigned to all stages on Mad and Geneva Creeks. This is high, but the obstruction of bridges could cause this variation. Stage-discharge data from Table A-4 were used for the Mississippi River. A standard error in stage of 0.5 foot was used for the Mississippi River.

(3) Levee Information. For each reach, the location where the levee would first overtop was identified. Since the existing levees meet Federal standards, the probable failure point equaled the levee crest elevation. For flood elevations above the crest, the exterior and interior water levels were assumed equal. If the computed damages for the reach were significant, the existing crest was increased in 1-foot increments to evaluate raising the levee. If computations revealed insignificant damage and high performance statistics, then raising the levee was not evaluated. The specific overtop locations are discussed in the paragraphs on results.

b. HEC-FDA Results. The computed results for each reach included the equivalent annual damage and the project performance for each alternative. Refer to Appendix B - Economic Analysis for information on equivalent annual damage and the economic selection of the recommended plan.

This section discusses only project performance. The long-term risk gives the probability of the levee being exceeded during a 10-, 25-, or 50-year period. Obviously, the longer the period, the greater the chance of the levee crest being exceeded. The conditional non-exceedance probability looks at performance by event. It gives the chance of the levee containing (not being overtopped by) the specified exceedance probability. Remember that with risk-based analysis, the .01 probability event is not one clearly defined stage; instead, it consists of a family of stages. In order for the Corps of Engineers to certify a levee for the Federal Emergency Management Agency, the 1% event must have conditional non-exceedance probability larger than .95. Levees can be certified without using risk if the levee crest is 3 feet above the water surface profile for the 1% event. This may appear confusing, but with risk-based analysis, each of the plotted profiles in the HEC-RAS section has a conditional non-exceedance of about 50%.

(1) Results for Reach 1: From Mouth to Upstream of 5th Street. The failure point for the existing levee is at the floodwall immediately downstream of the closure structure at 5th Street. The floodwall crest is elevation 559.4 feet NGVD. The stage discharge data were from the cross section 50 feet downstream of the 5th Street Bridge (ID#2158). After evaluating Alternative A (without-project), Alternative C (reservoirs or with-project), and Alternative D (without project but improved channel), the levee crest was increased in 1-foot increments. The first increment (crest elevation 560.4 feet NGVD) would require raising 1,000 linear feet of levee. The second increment (crest elevation 561.4 feet NGVD) would require raising the entire length of the levee. This is also true of the third increment (crest elevation 562.4 feet NGVD). The performance statistics for the alternatives appear in Table A-15. Alternatives B and C evaluate the reservoirs using the existing channel.

Table A-15. HEC-FDA performance statistics for Reach 1

	Long	Term	Risk	Conditional Non-Exceedance Probability by Events						Crest
Alt	**10-yr**	**25-yr**	**50-yr**	**10%**	**4%**	**2%**	**1%**	**.4%**	**.2%**	**Feet**
A+0	.059	.14	.26	.9999	.98	.93	.81	.58	.41	559.4
A+1	.044	.11	.20	1.00	.99	.96	.87	.69	.54	560.4
A+2	.032	.08	.15	1.00	1.00	.98	.93	.82	.72	561.4
A+3	.020	.05	.10	1.00	1.00	.99	.98	.94	.90	562.4
D+0	.024	.06	.11	1.00	1.00	.99	.96	.89	.83	559.4
D+1	.020	.05	.09	1.00	1.00	1.00	.99	.97	.96	560.4
D+2	.019	.05	.09	1.00	1.00	1.00	1.00	.99	.99	561.4
D+3	.019	.05	.09	1.00	1.00	1.00	1.00	1.00	1.00	562.4
B	.005	.01	.03	1.00	1.00	.99	.98	.93	.88	559.4
C B+1	.002	.005	.01	1.00	1.00	1.00	.99	.98	.97	560.4

(2) **<u>Results for Reach 2: From Upstream of 5th Street to Upstream of Washington Street</u>.** The failure point for the existing levee is the elevation of the railroad where it crosses Washington Street—at elevation 560.5 feet NGVD. The stage-discharge data were from the HEC-RAS model at the upstream face of the Washington Street Bridge. The performance statistics appear in Table A-16. The alternatives are identical to and defined in the Reach 1 section.

Table A-16. HEC-FDA performance statistics for Reach 2

	Long	Term	Risk	Conditional Non-Exceedance Probability by Events						Crest
Alt	**10-yr**	**25-yr**	**50-yr**	**10%**	**4%**	**2%**	**1%**	**.4%**	**.2%**	**Feet**
A+0	.08	.19	.34	.999	.97	.88	.72	.47	.31	560.5
A+1	.06	.14	.26	.999	.99	.93	.81	.60	.43	561.5
A+2	.04	.10	.19	1.000	.99	.96	.89	.73	.60	562.5
A+3	.03	.07	.13	1.000	.998	.98	.95	.88	.81	563.5
D+0	.05	.13	.25	.999	.99	.93	.82	.62	.46	560.5
D+1	.04	.09	.17	1.000	.99	.97	.91	.78	.67	561.5
D+2	.02	.05	.10	1.000	.998	.99	.97	.92	.87	562.5
D+3	.02	.05	.09	1.000	.999	.998	.99	.98	.97	563.5
B	.02	.04	.08	1.000	.998	.98	.95	.84	.74	560.5
C B+1	.005	.01	.03	1.000	.999	.99	.98	.93	.88	561.5

(3) <u>Results for Reach 3: Mad and Geneva Creeks at Existing Heinz Plant</u>. The failure point of the existing system is immediately downstream of the Heinz access road closure. This point, 485 feet upstream from the mouth of Geneva Creek, is 10 feet downstream of the access road bridge (cross section 0.6) and has a crest at elevation 572.35 feet NGVD. The performance statistics for the existing levee appear in Table A-17. Since the conditional non-exceedance probability is 99.9% for the 1% event with little damage, no alternatives were examined.

Table A-17. HEC-FDA performance statistics for Reach 3

Alt	Long 10-yr	Term 25-yr	Risk 50-yr	Conditional Non-Exceedance Probability by Events 10%	4%	2%	1%	.4%	.2%	Crest Feet
A+0	.019	.048	.932	1.0000	1.0000	1.0000	0.9999	0.9998	0.9997	572.35

(4) <u>Results for Reach 4: Mississippi River</u>: The top of the levee and flood wall along the Mississippi River are at elevation 559.5 feet NGVD and will be overtopped at the same time. The performance statistics for the existing levee (D+0) appear in Table A-18. Since the conditional non-exceedance probability is 89% for the 1% event, the levee crest was increased in 1-foot increments. Unfortunately, increasing the crest 3 feet produced so little damage that the model became unstable. For this reason, the third increment increased the crest only 2.3 feet.

Table A-18. HEC-FDA performance statistics for Reach 4, Mississippi River

Alt	Long 10-yr	Term 25-yr	Risk 50-yr	Conditional Non-Exceedance Probability by Events 10%	4%	2%	1%	.4%	.2%	Crest Feet
D+0	.05	.12	.22	1.000	0.999	0.991	0.89	0.51	0.23	559.5
D+1	.02	.04	.09	1.000	1.000	0.999	0.98	0.86	0.73	560.5
D+2	.0028	.0071	.014	1.000	1.000	1.000	0.9996	0.996	0.993	561.5
D+2.3	.0028	.0069	.013	1.000	1.000	1.000	0.9997	0.998	0.997	561.8

c. <u>HEC-FDA Consequences of Failure to Close Gate</u>. The consequences of failing to close an opening were analyzed by correlating the exterior river stage to the interior flood stage (reference 14). Where there was a choice, assumptions that would produce the highest interior stage were adopted. The relationship between exterior and interior stage was used to estimate damage. This section discusses how the exterior versus interior relationship was developed. There are no risk performance statistics for this section.

(1) <u>Reach 1, 2nd Street Bridge</u>. During a storm in 1990, water entered through 5^{th} Street before the opening could be blocked. This was the only time this has happened along Mad Creek since the project was completed. This event was used to compute the probability of the closure not being made (1/30=.03). The consequence of not closing the opening at 2nd Street was analyzed by relating the exterior stage of Mad Creek to an interior water level if the gate was left open. The interior stage was determined by estimating the volume of water flowing through the opening. Damages were estimated assuming closure would be completed 97% of the time.

Inflow starts at zero when Mad Creek rises to the sill elevation, increases until the exterior stage peaks, and then returns to zero as the exterior stage falls below the sill elevation. The sill is at elevation 553 feet NGVD and the levee crest is at elevation 559.5 feet NGVD. The width between abutments is 59.3 feet. From an examination of the rating curve at the bridge, a discharge of 10,000 cfs produces a stage of 553 feet NGVD. The amount of time the discharge is above 10,000 cfs was obtained from the hydrograph of the .002% chance storm. Inflow will occur for about 3 hours. To simplify the computations, a triangular stage hydrograph was used for various exterior elevations. The exterior stage started at elevation 553 feet, reached a peak in 1.5 hours, and then returned to elevation 553 feet in 3 hours. The interior stage was computed by routing the inflow into the elevation-volume curve for the protected area. Unfortunately, the elevation-volume had to be estimated using just a few known points.

Table A-19. Elevation-area-volume relationship for interior area at 2nd Street

Interior Elevation Feet NGVD	Estimated Area Acres	Estimated Volume Ac-ft
550	0	0
551	5.0	1.2
552	10.7	9.1
553	20.0	24.4
554	25.0	16.9
555	28.5	73.7
556	31.5	103.7
557	34.0	136.4
558	36.0	171.4
559	37.2	208.0
560	38.4	245.8

A spreadsheet developed to compute the time required to fill a protected area if the levee failed was used to estimate interior stage. The program used the weir equation to compute flow into the protected area using 5-minute computation intervals to compute the increase in interior stage. Computations used a weir length of 59.3 feet and a weir coefficient of 2.75. The highest interior water level always occurred after the exterior water level had peaked. The maximum interior level occurred when the falling exterior stage equaled the rising interior stage. The relationship between exterior and interior stage appears in Table A-20.

Table A-20. Transform from exterior stage to interior stage at 2nd Street

Exterior Elevation Feet NGVD	Interior Elevation Feet NGVD
553	550.0
554	552.4
555	553.8
556	554.9
557	555.8
558	556.8
559.5	558.2

(2) Reach 3, Isett and Service Road Openings. Over the life of the project, water has entered the openings on Geneva Creek twice. In 1990, a fence blocked the service road bridge and water entered both openings. In 1993, the Mississippi River was high when another storm overtopped both sills. Because the ground slopes from Geneva to Mad Creek within the interior, the water was not trapped. Instead, it flowed at shallow depth toward Washington Street, re-entered Mad Creek, and caused slight damage.

Because of the brief response time and the unlikelihood that sandbags would be placed in time, the closures were analyzed under the assumption that both openings always would be open. The estimate of interior water levels assumed that all inflow was trapped. Even so, because the sills are high and small, the computed interior water level is significantly lower than the exterior water level.

The sill of the service road is elevation 567.8 feet NGVD with a width of 42 feet, which increases to 63 feet at the levee crest (elevation 572.35 feet NGVD). The sill of Isett Avenue is elevation 586.5 feet NGVD with a width of about 60 feet. Under normal depth conditions on Geneva Creek, a discharge of 4,000 cfs overtops both sills. The duration of an overtopping event is about 1 hour based on the hydrograph of the 0.2% chance storm. In relating exterior to interior water levels, it was assumed that for all exterior stages above 567.7 feet, the inflow peaked in .5 hour and returned to zero after 1 hour. The interior is about 1,800,000 square feet (41 acres). The inflow volume was estimated by using twice the calculated inflow through the service road. The discharge was computed using the weir equation with a coefficient of 2.75 and a length of 63 feet. The relationship between exterior and interior levels used in HEC-FDA appears in Table A-21.

Table A-21. Transform from exterior stage to interior stage at Service Road

Exterior Elevation Feet NGVD	Interior Elevation Feet NGVD
567.7	567
568.7	567.3
569.7	568.0
570.7	568.8
571.7	569.8
572.3	570.4

10. FLOOD WARNING

a. Basin Characteristics. Discharge hydrographs for both Geneva and Mad Creek appear on plate A-10. These HEC-HMS plots were produced by applying 4 inches of rain during 1 hour. Since the time between the center of mass of rainfall and the peak discharge is only 1 to 2 hours, the largest warning time will probably be about a half hour. If the time between bursts of rainfall is longer than 6 to 8 hours, the runoff will form two separate events and will not be additive. Applying the synthetic rainfall in 1 hour produces higher peaks than periods of 2 or 3 hours. However, in real life the total of the past 3 hours will probably be the most important period to monitor for the Mouth of Mad Creek.

In an effort to define a relationship between rainfall and peak discharge, the without-project HEC-HMS model was used to compute peak discharges. It was produced by applying a series of 1-hour storms for rainfall amounts ranging from 1 to 4 inches. This information appears on plate A-11 for the mouth of Geneva and Mad Creeks. Unfortunately, the relationship between rainfall and runoff is complicated by the ability of the soil to absorb moisture. The Soil Conservation Service addresses this problem by totaling the rainfall falling in the 5 days before the storm (reference 15). This is then used to adjust the amount of rainfall absorbed by the soil. Table A-22 is based upon this approach; it arbitrarily creates three classes of antecedent moisture conditions. A curve for each class appears on plate A-11.

Table A-22. Total 5-day antecedent rainfall in inches

Condition	Dormant Season	Growing Season
Dry	Less than 0.5 inches	Less than 1.4 inches
Average	0.5 to 1.1 inches	1.4 to 2.1 inches
Saturated	Over 1.1 inches	Over 2.1 inches

Low temperatures prevent evaporation and prolong saturated conditions; this introduces error into Table A-22. There is no precise adjustment for temperature. Nevertheless, one should be aware of the temperature during the previous 5 days to evaluate the flood threat.

The plots on plate A-11 can be used with target discharges at various damage centers to determine preliminary alarm stages. The information can also be used to trigger alert, mobilization, and closure actions. Table A-23 lists information on closures for the project. The information came from reference 16 or recent surveys.

Table A-23. Information on closures

Location	Closure Type	Sill Elevation Ft NGVD		Approx. Discharge cfs	Approx. Frequency Exceedance
RR Closure 600 ft south of Washington Avenue Mad Cr.	Sandbag	559.55			
2nd Street closure Mad Cr.	Gate	553.0		10,000	See text
Isett Ave closure on Geneva Cr.	Sandbag	568.5		4,250	Below .002
Heinz service road closure on Geneva Cr.	Sandbag	567.8		4,500	Below .002

Given the short response time on Geneva Creek, it is unlikely that any type of closure, let alone sandbags, could be placed in time. However, sandbags could be used on Geneva Creek in situations where a high Mississippi River could make overtopping the sill elevation more likely.

Most of the time, the peak discharge on Mad Creek will determine when to close 2nd Street. The low steel of the bridge varies from elevation 549.7 to 551.7 feet NGVD. Under most conditions, a discharge of 9,000 cfs will pass under the bridge. However, when the water level at the mouth of Mad Creek is higher than elevation 547 feet NGVD, the discharge required to touch the low steel decreases. Plate A-12 shows a family of rating curves at 2nd Street Bridge for various starting water levels. Stage duration data for the Mississippi River show that 95% of the days of the year (on average) the water surface will be below elevation 546.5 feet NGVD (547 feet 1912 datum) at the mouth of Mad Creek. Between a starting water level of 547 feet NGVD and 550.5 feet NGVD, the target discharge decreases from 9,000 to 4,000 cfs.

b. Flood Warning System. A replacement flood warning system for Mad Creek was designed under contract; it is estimated to cost $72,000. The system uses three recording rainfall gages equipped with programmable logic controllers and data transmission devices that convey a UHF signal to the Public Safety Building. There a computer stores and monitors data and signals warnings. The system was to facilitate the frequent closure of 5th Street. The report, drawings, and specifications are on file at the Rock Island District office (reference 8). It now appears that 5th Street will be raised, eliminating the closure. A Value Engineering Team will reevaluate the design prior to the completion of the final Detailed Project Report.

(1) Description of System Components. The flood potential is evaluated from real time precipitation at the three sites (see plate A-1). Two gages are in the upper Mad Creek basin. Gage 1 is off Route 38, near the Municipal Golf Course. Gage 2 is near 2900 180th Street, just west of Route 61. Gage 3 is in the Geneva Creek basin, near the water tower, southeast of the intersection of Bypass 61 and Bidwell Avenue. A tipping bucket style gage is recommended with a collector diameter of 8.625 inches. This gage has resolution of .01 inch (0.25 mm) and an accuracy of 0.5% at 0.5 inch (12.50 mm) per hour. The gage can be heated to record the water equivalent of snowmelt if year-round operation is chosen.

A programmable logic controller near the rain gage receives a signal from the tipping bucket rain gage for each 0.01 inch of rainfall at the gage. The controller calculates the precipitation for the previous 0.5-hour, 1-hour, 2-hour, 3-hour, and 6-hour periods. The controller attaches a time increment (preferred setting of 1 minute) to the incremental precipitation and calculates total rainfall, as well as daily rainfall.

The logic controller sends the information by radio modem to a proposed supervisory control and data access system (SCADA) located at the Public Safety Building. A new computer will compare real time precipitation with trigger values and send alerts when the values are exceeded. Interface software on the computer manages the precipitation record and other control data. It maintains quarter-hour records of precipitation for each gage on a daily basis using military time. The recommended length of record is 18 months. The format of the recorded data will be such that it can be loaded directly into a standard spreadsheet program.

The new computer also receives data from an ultrasonic level detector located on the 5th Street Bridge. The stage data will provide an alert for high water at the 5th Street Bridge. The unit has a resolution 0.01 foot with an accuracy of 0.5% of its calibrated span. The maximum span is 60 feet. The gage will be zeroed at 545.0 MSL. A stage gage should also be installed on the bridge to make annual calibration easier. The stage gage has a programmable logic controller and transmitter as part of the electronics package.

(2) Computer Handling of Precipitation Data and Generation of Alarms. The computer will maintain a real time record of precipitation depth for each gage. It will also compute the total basin rainfall using Thesian polygons for the same time increments, compare totals to preset rainfall depths, and generate an alarm signal, if appropriate. The computer will make comparative analysis of the rainfall intensity for the gages to the following data and signal an alarm when any value is exceeded. The comparable values can be set through the computer interface and keyboard. Initial values recommended by the contractor appear in Table A-24.

Table A-24. Initial values for rainfall alarms within Mad Creek basin

0.5 hour:	2 inches per hour
1 hour:	1.5 inches per hour
2 hour:	0.75 inches per hour
3 hour:	0.5 inches per hour
6 hour:	0.25 inches per hour

The computer will indicate real time stage data for Mad Creek. The stage gage data will be stored in 15-minute increments for at least 18 months. The record form will include date and military time. The storage data format will be loadable to standard spreadsheet programs. The computer analyzes the stage data and signals the central alarm if values in Table A-25 are exceeded. The two stage values recommended by the contractor are intended to provide approximate 90-minute and 60-minute warnings of a flood level on Mad Creek.

Table A-25. Stage alarms on Mad Creek

Stage	Elevation	Comment
3.5	548.5 MSL	14-Year Freq.
5.9	550.9 MSL	45-Year Freq.

(3) <u>Maintenance for Flood Warning System</u>. Maintenance of the rain gages includes annual cleaning and re-zeroing of the precipitation accumulator. The local sponsor should also plan on inspecting each gage once every 3 months during flood season. Maintenance of the stage gage includes re-zeroing and periodic discharge measurements to calibrate discharge to stage. This is not essential, but it would allow the collection of annual peak stage and discharge for Mad Creek that will decrease the uncertainty of the rainfall-runoff relationship.

Maintenance of the data transmission system will be minimal. The system should be checked annually for damage.

Maintenance of the computer records requires periodically backing up files.

After an emergency, as conditions permit, the superintendent will initiate a general cleanup of all flood control facilities, make a general inspection of the project, and repair all damage to the project works. Demobilization of flood control activities will include the release of emergency personnel, an inventory of equipment and supplies, and cleaning, storing, and replenishing equipment and supplies. Procedures will then revert to ordinary inspection and maintenance.

In addition to the semi-annual reports, the superintendent will prepare post-flood reports after significant floods and forward one copy to the District Engineer. The report will be a complete flood history and will include a log of operations, a daily tabulation of river stages, a discussion of pertinent factors in operating and maintaining the project, and any other useful information. Operation and maintenance factors will include problems encountered, weather conditions encountered (including effects of ice on operation), damage incurred, repairs required, and other significant factors which occurred during the operation and maintenance of the project during the flood period. The report will also include a summary of the numbers, time, and cost of manpower and the quantities and costs of supplies and equipment that the protective effort required. The flood report can be useful in future flood fights.

11. REFERENCES

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5. *Mad Creek, City of Muscatine, Flood Mitigation Preliminary Investigation*, U.S. Department of Agriculture, Natural Resources Conservation Service, August 1996.

6. *HEC-HMS Hydrologic Modeling System, Users Manual and Technical Reference Manual*, Corps of Engineers, Hydrologic Engineering Center, Davis, California, January 2000.

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10. *Time of Concentration and Storage Coefficient Values for Illinois Streams*, J. Graf, U.S. Geological Survey, Urbana, Illinois, March 1982.

11. *HEC-RAS River Analysis System Hydraulic Reference Manual Version 3.0*, Corps of Engineers, Hydrologic Engineering Center, Davis, California, January 2001.

12. *HEC-FDA Flood Damage Reduction Analysis User's Manual Version 1.0*, Corps of Engineers, Hydrologic Engineering Center, Davis, California, January 1998.

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14. Closure and Interior Facilities *for Levee Projects: Principles, Case Examples, and Risk Based Analysis Concepts Version 1.0*, Corps of Engineers, Hydrologic Engineering Center, Davis, California, September 1996.

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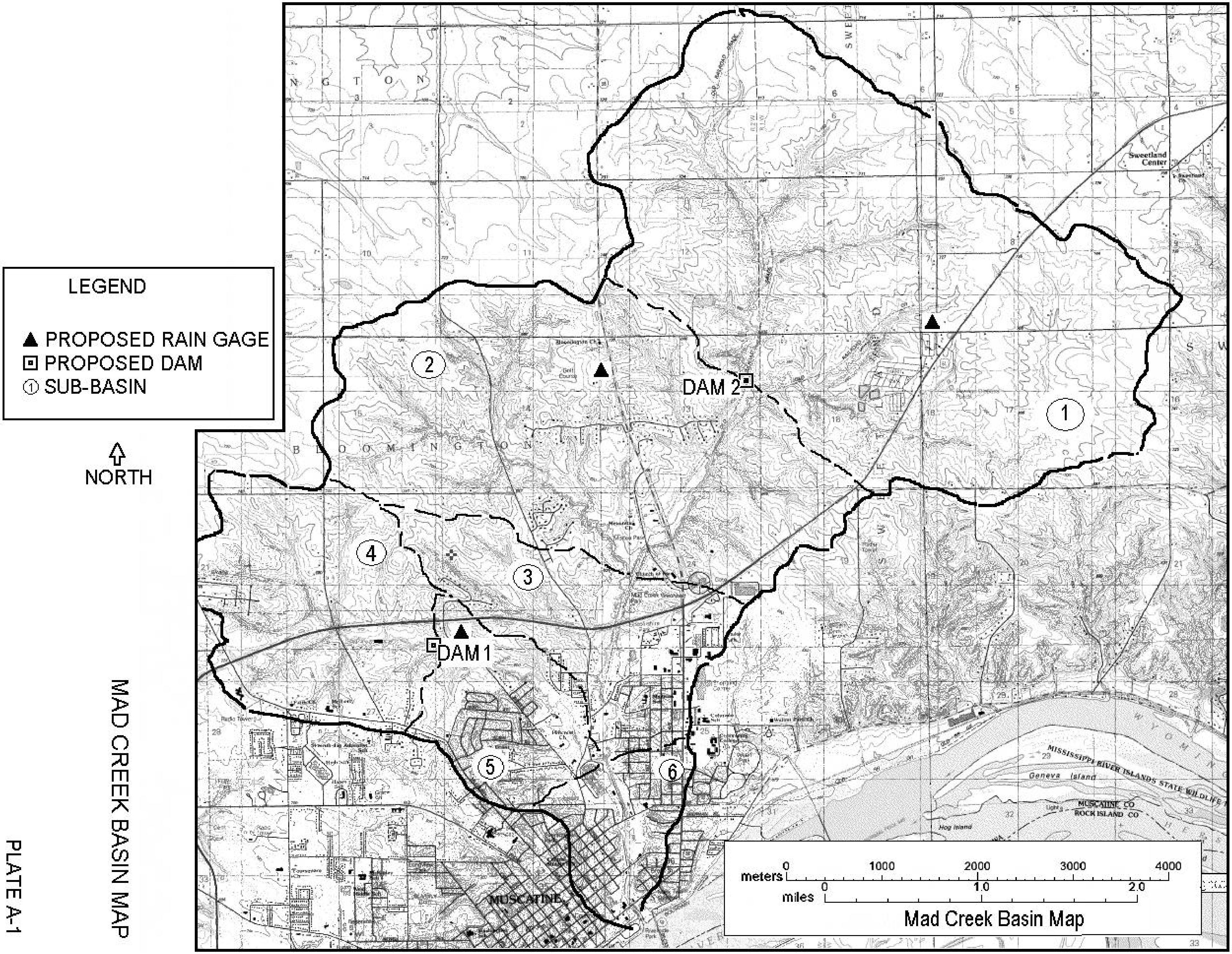
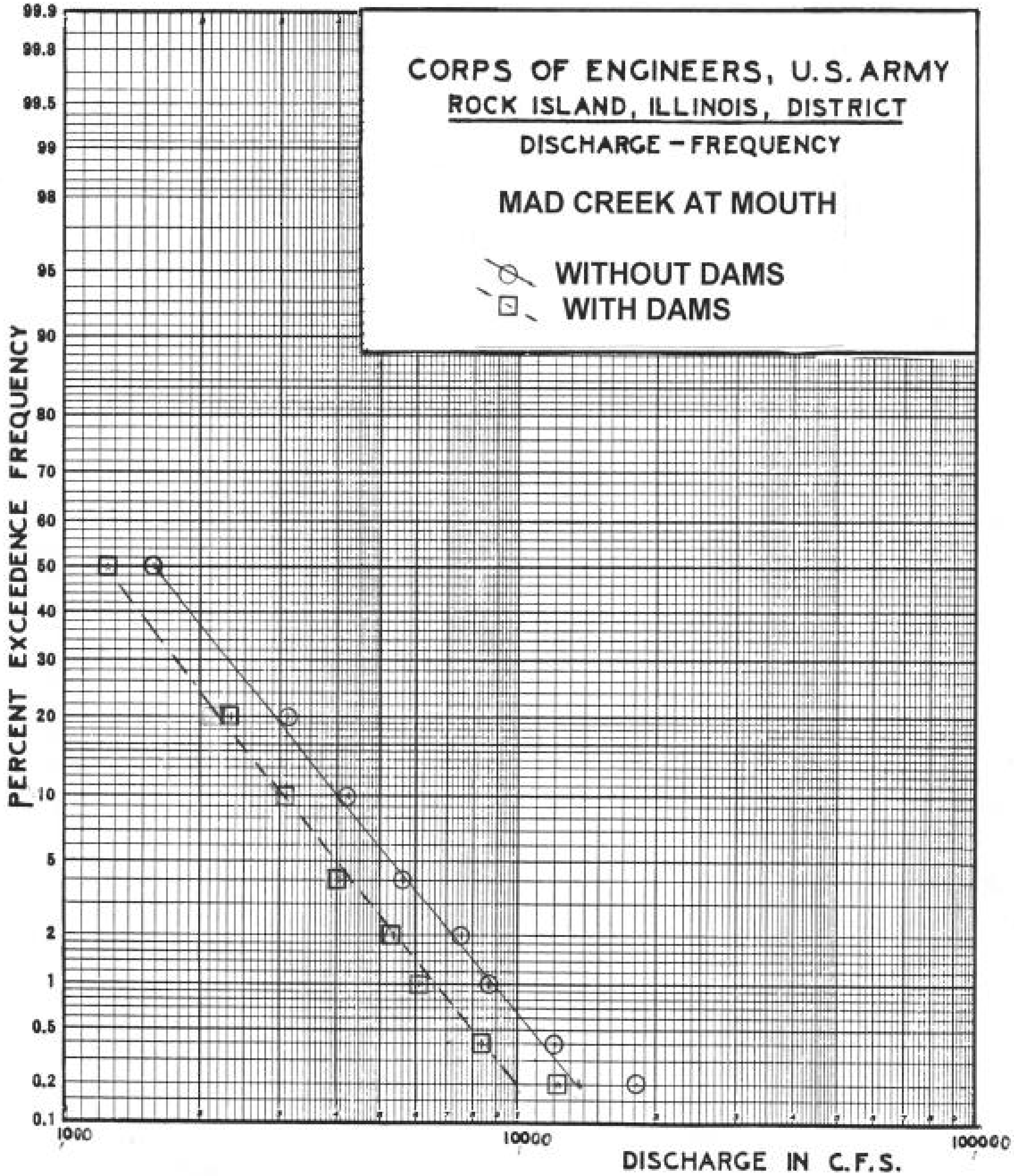


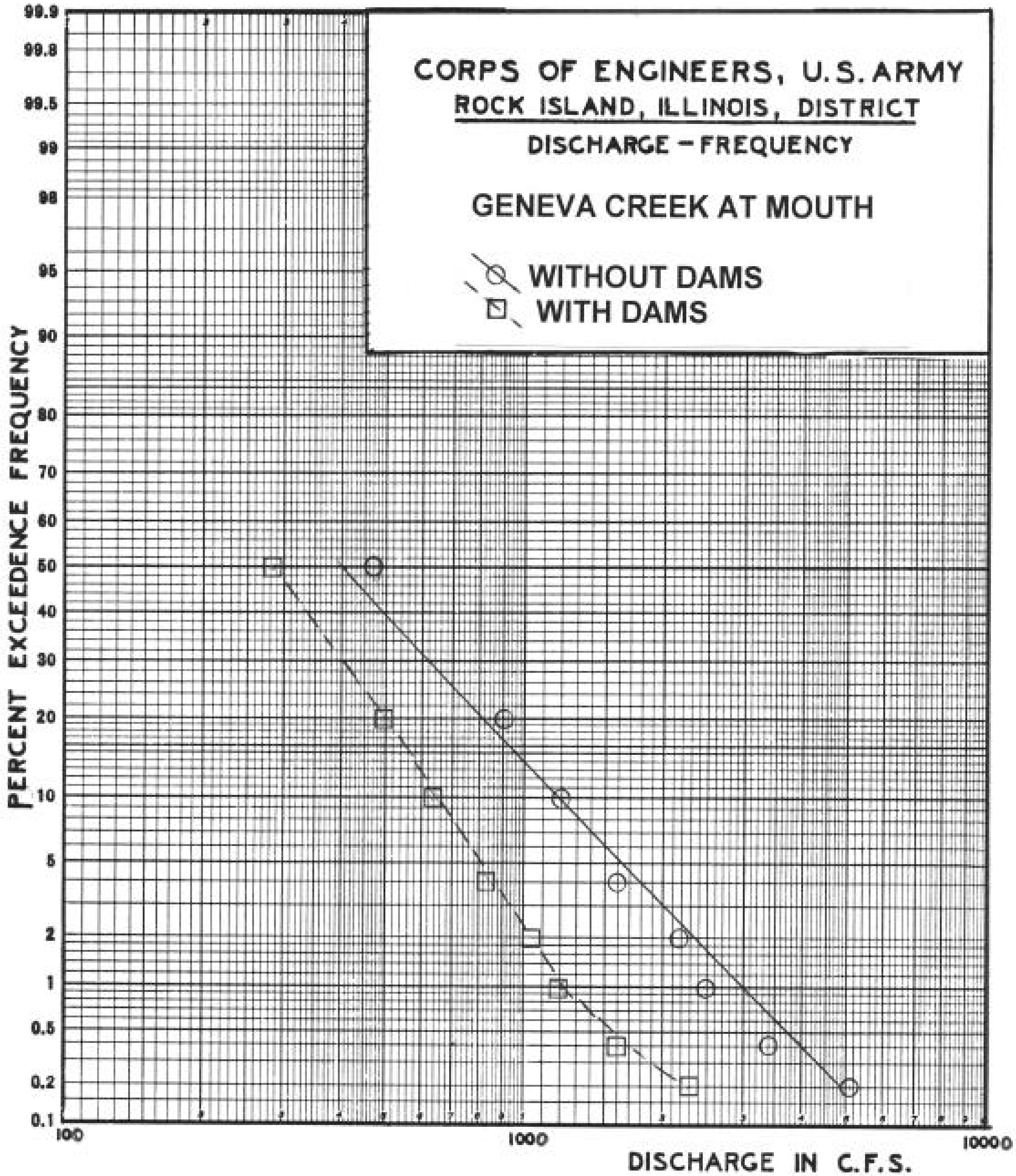
MAD CREEK BASIN MAP

PLATE A-1



CORPS OF ENGINEERS, U.S. ARMY
ROCK ISLAND, ILLINOIS, DISTRICT
DISCHARGE – FREQUENCY
MAD CREEK AT MOUTH
WITHOUT DAMS
WITH DAMS
PERCENT EXCEEDENCE FREQUENCY
99.9
99.8
99.5
99
98
95
90
80
70
60
50
40
30
20
10
5
2
1
0.5
0.2
0.1
1000
10000
100000
DISCHARGE IN C.F.S.

PLATE A-2



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ROCK ISLAND, ILLINOIS, DISTRICT
DISCHARGE – FREQUENCY
GENEVA CREEK AT MOUTH
WITHOUT DAMS
WITH DAMS
PERCENT EXCEEDENCE FREQUENCY
99.9
99.8
99.5
99
98
95
90
80
70
60
50
40
30
20
10
5
2
1
0.5
0.2
0.1
100
1000
10000
DISCHARGE IN C.F.S.

PLATE A-3

Mad Creek Water Surface Profiles Alternative- A (Existing)

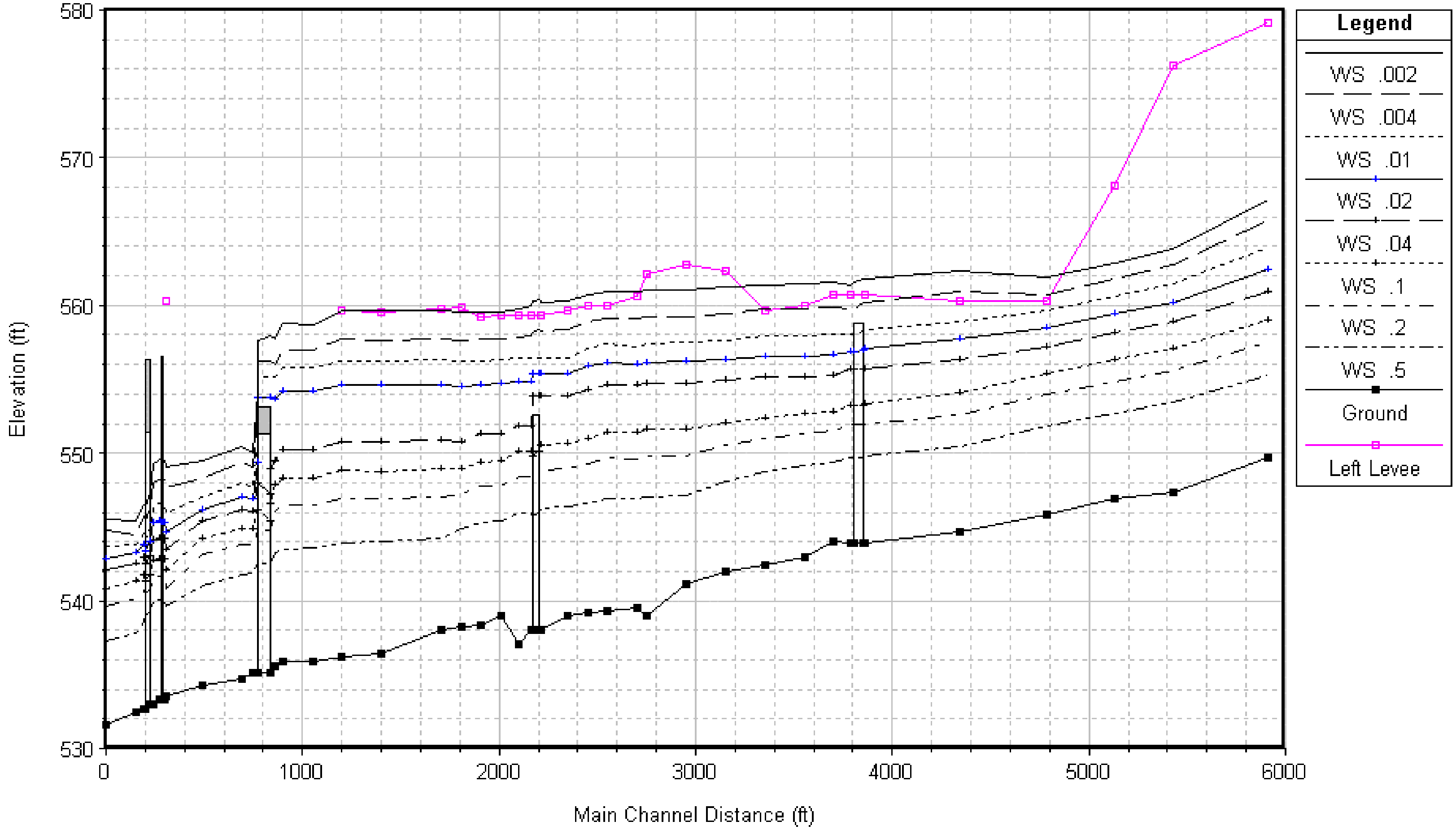


PLATE A-4

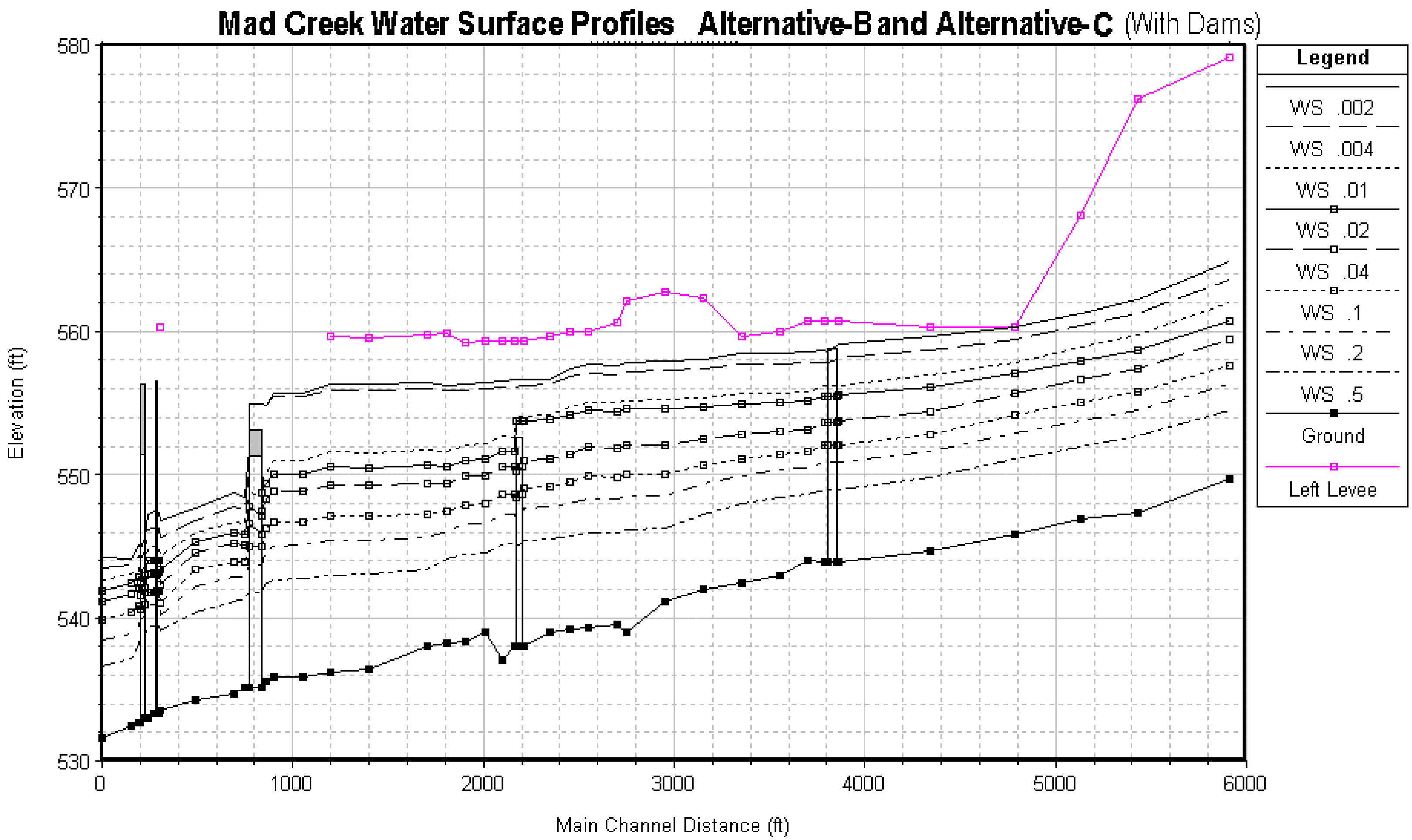
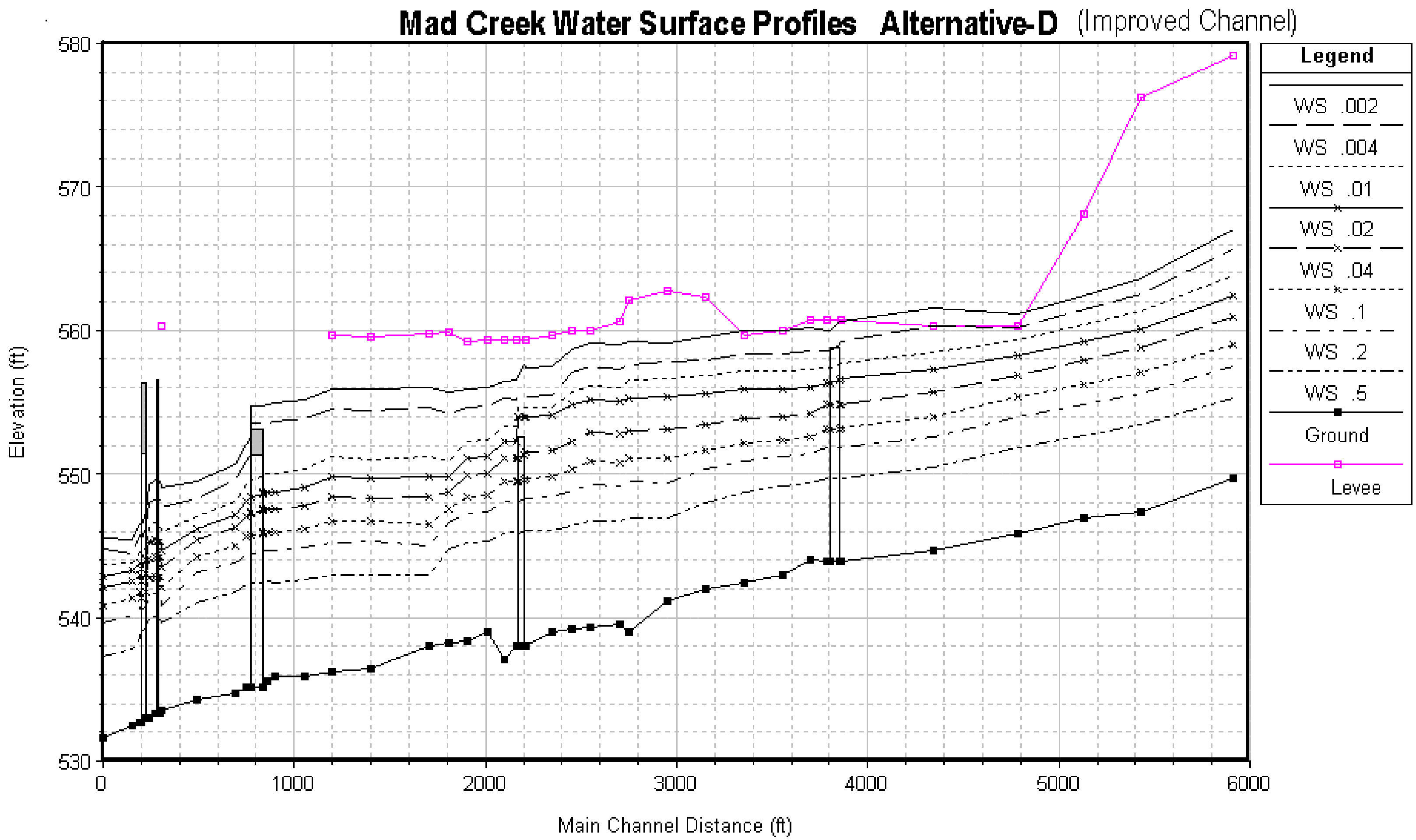

Mad Creek Water Surface Profiles Alternative-B and Alternative-C (With Dams)
Elevation (ft)
Main Channel Distance (ft)
580
570
560
550
540
530
0
1000
2000
3000
4000
5000
6000
Legend
WS .002
WS .004
WS .01
WS .02
WS .04
WS .1
WS .2
WS .5
Ground
Left Levee

PLATE A-5



Mad Creek Water Surface Profiles Alternative-D (Improved Channel)
Legend
WS .002
WS .004
WS .01
WS .02
WS .04
WS .1
WS .2
WS .5
Ground
Levee
Elevation (ft)
580
570
560
550
540
530
0
1000
2000
3000
4000
5000
6000
Main Channel Distance (ft)

PLATE A-6

4/06/01

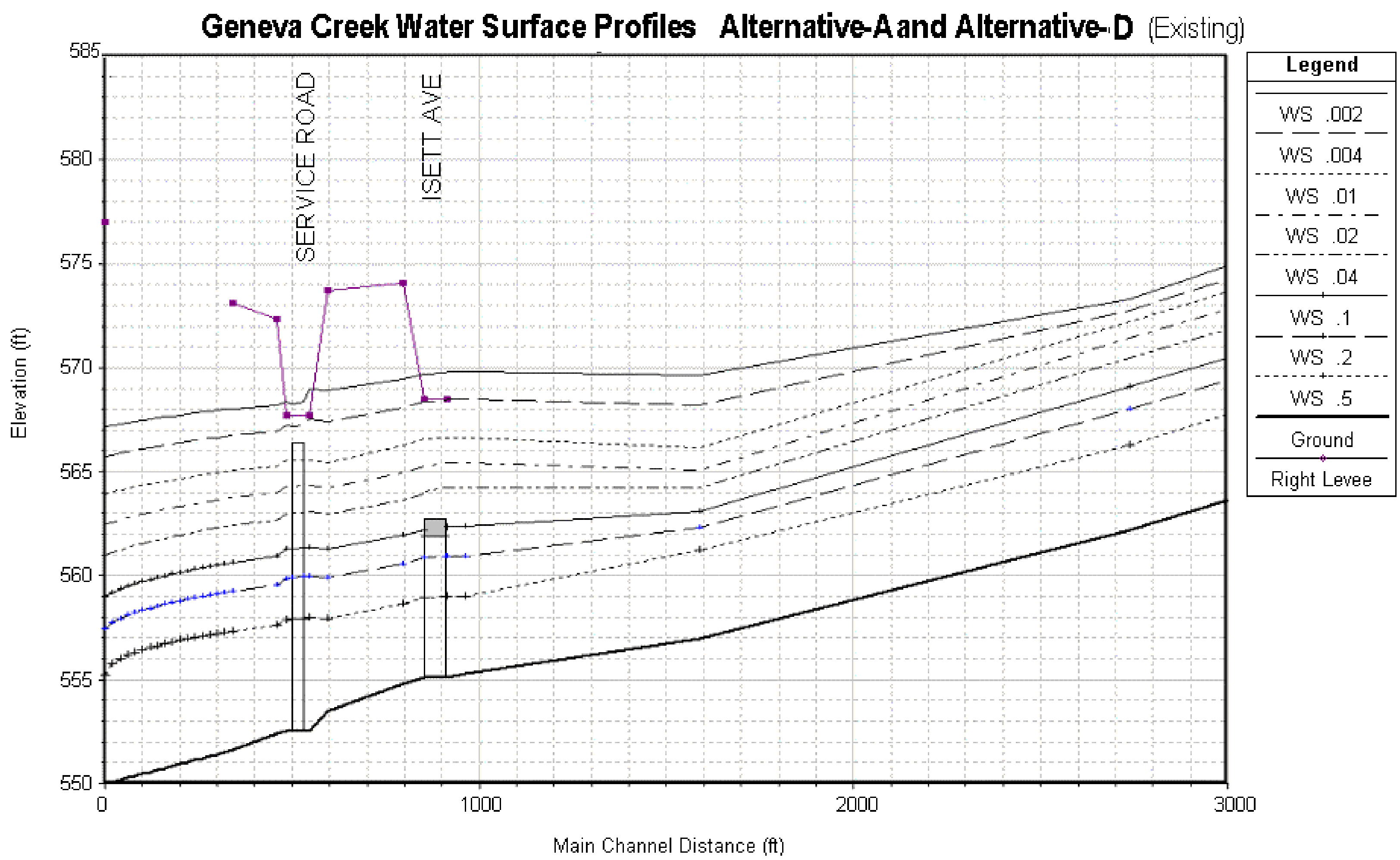


PLATE A-7

Geneva Creek Water Surface Profiles Alternative-B and Alternative- C (With Dams)

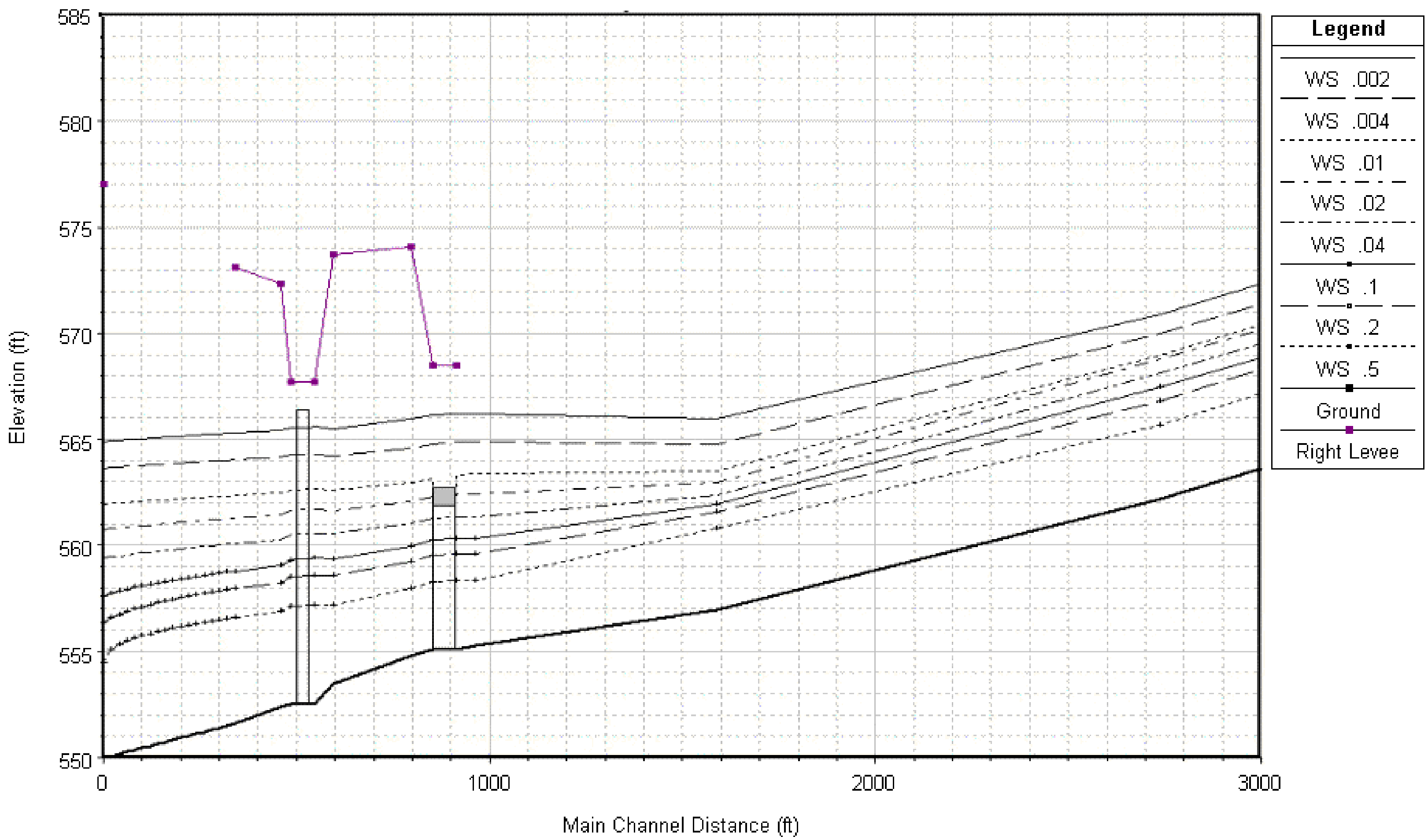
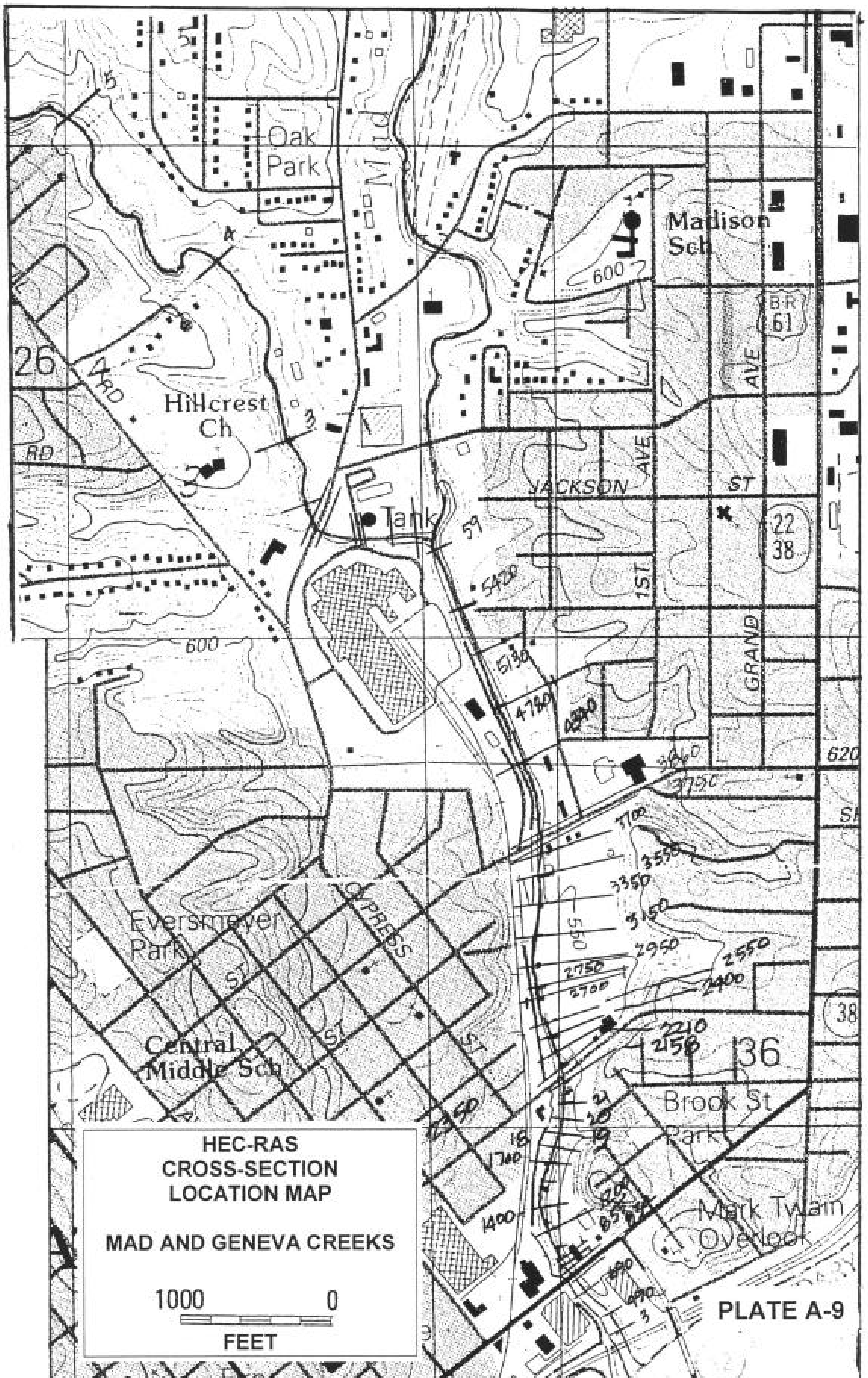


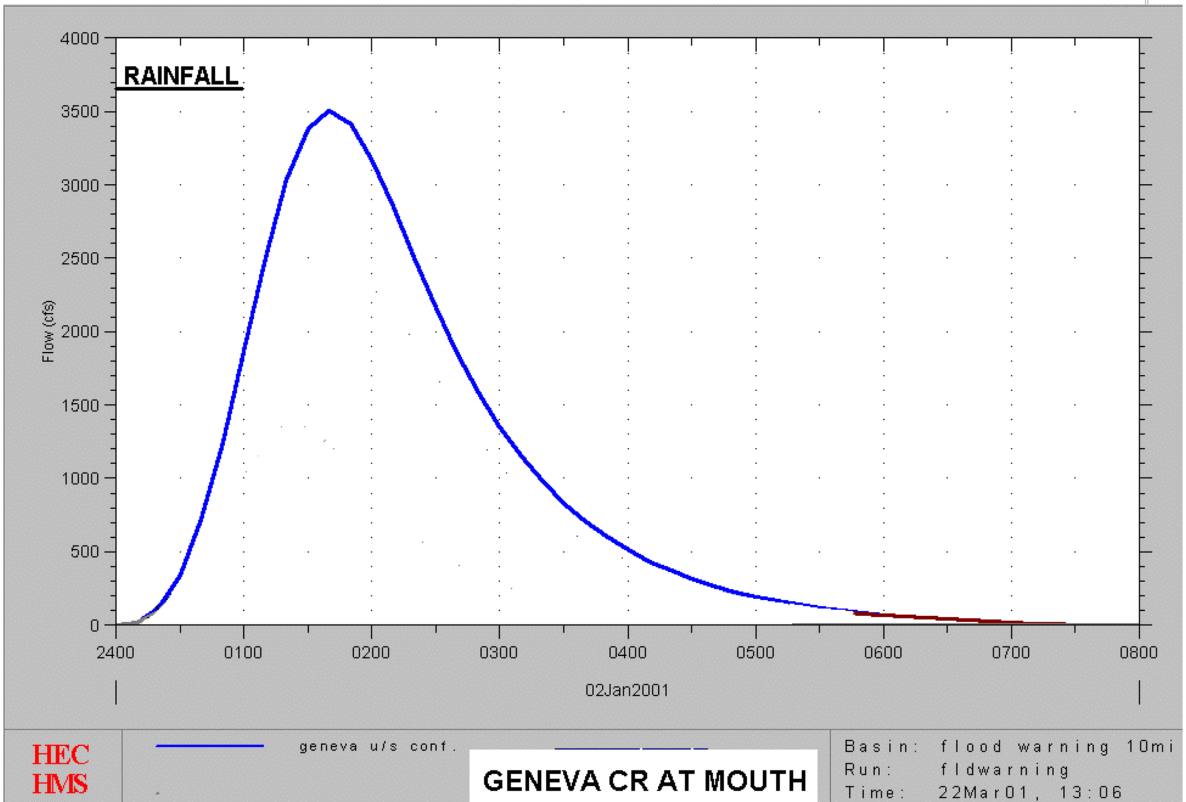
PLATE A-8



HEC-RAS CROSS-SECTION LOCATION MAP

MAD AND GENEVA CREEKS

PLATE A-9


RAINFALL
4000
3500
3000
2500
2000
1500
1000
500
0
Flow (cfs)
2400
0100
0200
0300
0400
0500
0600
0700
0800
02Jan2001
HEC HMS
geneva u/s conf.
GENEVA CR AT MOUTH
Basin: flood warning 10mi
Run: fldwarning
Time: 22Mar01, 13:06

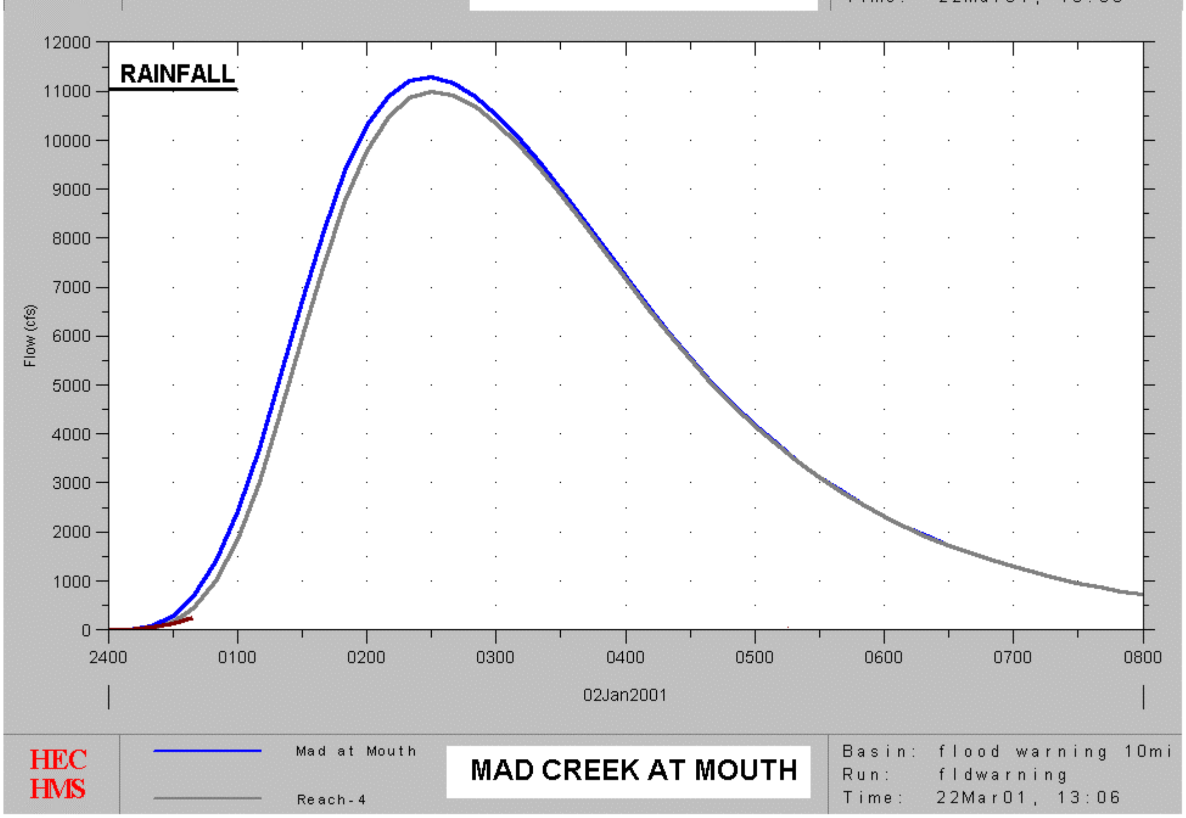
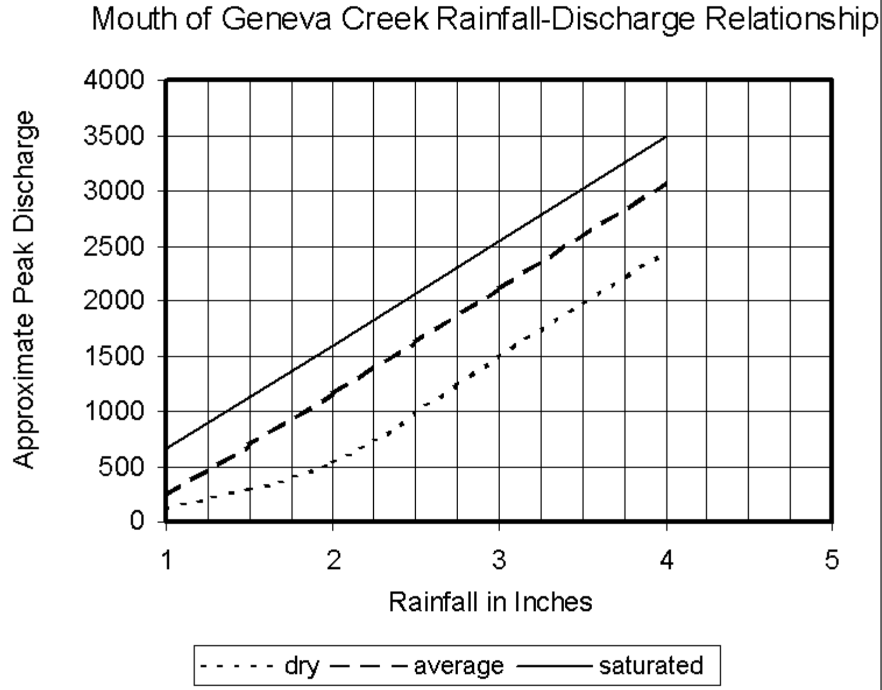
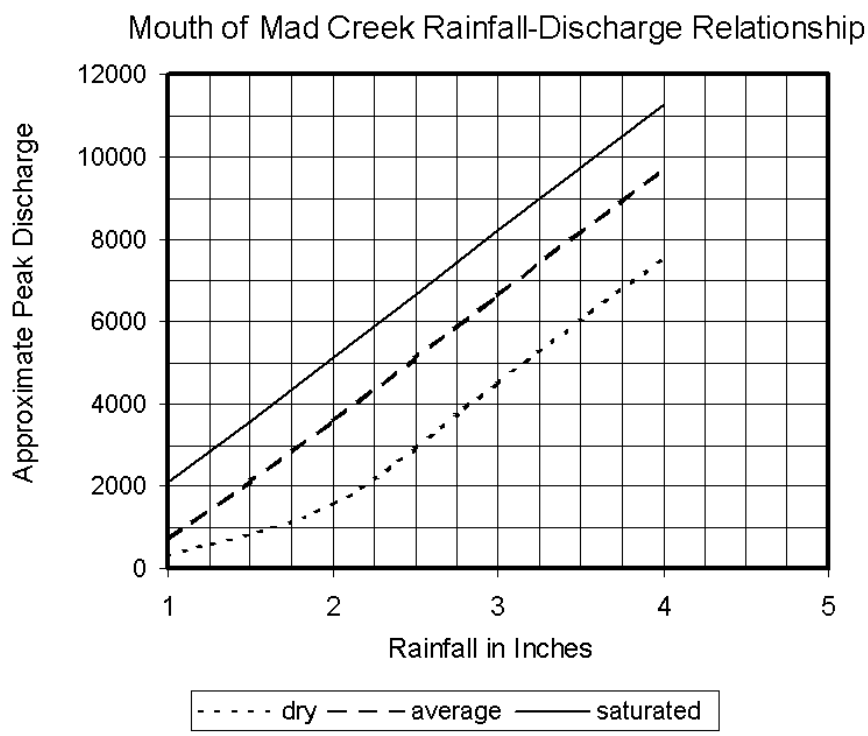

RAINFALL
12000
11000
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
Flow (cfs)
2400
0100
0200
0300
0400
0500
0600
0700
0800
02Jan2001
HEC HMS
Mad at Mouth
Reach-4
MAD CREEK AT MOUTH
Basin: flood warning 10mi
Run: fldwarning
Time: 22Mar01, 13:06

PLATE A-10



Mouth of Geneva Creek Rainfall-Discharge Relationship
Approximate Peak Discharge
4000
3500
3000
2500
2000
1500
1000
500
0
1
2
3
4
5
Rainfall in Inches
dry
average
saturated



Mouth of Mad Creek Rainfall-Discharge Relationship
Approximate Peak Discharge
12000
10000
8000
6000
4000
2000
0
1
2
3
4
5
Rainfall in Inches
dry
average
saturated

Plate A-11

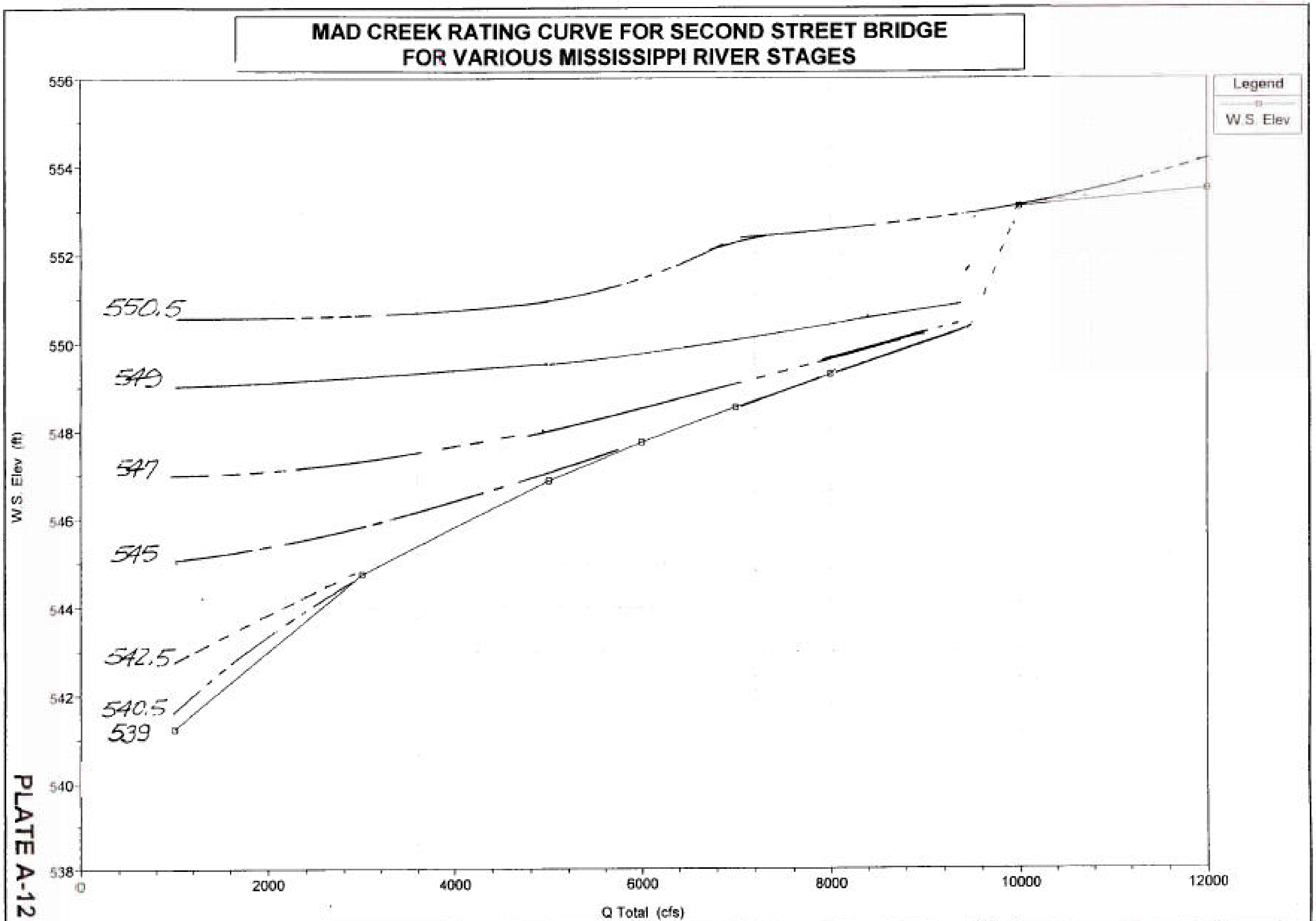

MAD CREEK RATING CURVE FOR SECOND STREET BRIDGE
FOR VARIOUS MISSISSIPPI RIVER STAGES
Legend
W.S. Elev
550.5
549
547
545
542.5
540.5
539
556
554
552
550
548
546
544
542
540
538
W.S. Elev (ft)
0
2000
4000
6000
8000
10000
12000
Q Total (cfs)

PLATE A-12